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RISK-BENEFIT ANALYSIS IN COAL MINE ROOF SUPPORT DESIGN USING STOCHASTIC MODELLING TECHNIQUE

Hongkui Gong¹, Ismet Canbulat², Mehmet S Kizil¹, Anna Mills³ and Jason Emery³

ABSTRACT: Various roof support design methodologies have been used in Australian coal mines, which include analytical, numerical and empirical models. These models are mainly based on the deterministic approach in which a single factor of safety is calculated for the roof support design. The main limitation of this design methodology is that it fails to account for the inherent variations existing in rock mass properties and other roof reinforcement elements. To overcome this issue, an improved design methodology based on stochastic approaches has been developed in which both the design inputs parameters and the outcomes (i.e., factor of safety) are expressed as probability distribution functions. This paper focuses on the application of stochastic modelling technique to evaluate the underground roof support strategies currently used in an underground coal mine located in the Bowen Basin. The starting point of the analysis is the existing analytical roof support models that identified the relevant design inputs in consideration. Based on the best fit probability distributions of input parameters determined by goodness of fit tests, a risk based design is conducted to quantitatively evaluate the risk of roof fall fatality under specific roof support system by using the probability of failure from Monte Carlo simulation and the associated underground personnel exposure.

INTRODUCTION

Roof strata control is one of the most critical components of underground coal mining. Without proper reinforcement, the roof strata may be destabilised resulting in catastrophic consequences to the health and safety of employees and significant financial loss due to the production downtime. It is widely accepted that in an underground environment rock mass properties and support elements can vary significantly within a short distance; roof stability is strongly dependent on these varying properties. Traditional roof support design for underground coal mine are primarily based on deterministic approaches in which the inputs parameters are presented as single values. Although such approaches provide a straightforward design process and the design outcome can be easily evaluated against the long established design criteria (i.e. factor of safety), they are unable to account for uncertainties governing the roof support performance in a quantifiable manner. In order to address this issue, the application of the stochastic modelling technique has been proposed. Stochastic modelling simply allows for the randomness of the input parameters in the roof support design.

In a stochastic design approach, the input parameters are expressed as probability distribution functions rather than single values. The design outputs (i.e. factor of safety) are also statistically distributed, based on which the probability of failure (POF) for a given roof support design can then be calculated. As such, the associated risk from the varied design inputs can be quantified, which in turn assists geotechnical engineers and mine management with the risk-based decision-making.

An improved stochastic approach will directly contribute to better risk assessment and management in underground roof support (Brown, 2012). This design methodology potentially accounts for all sources of inherent geotechnical uncertainties and field investigation errors and can enable geotechnical engineers to produce a risk-based roof support design for underground roadways. The decision-making process with respect to many of the risk-based problems such as potential fatality analysis and evaluation of roof support design against the relevant safety standards can be improved by representing risk quantitatively in terms of the probability of failure and the associated underground workforce exposure.

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UNDERGROUND ROOF SUPPORT DESIGN

Roof behaviour in underground coal mines

For the purpose of designing and implementing effective strata control strategies in underground coal mines, the overall design methodology is to obtain an improved understanding of the roof behaviour of laminated, weak coal mine strata. The stability of roadways in underground coal mines is vulnerable to two major causes: the mining induced stress redistribution around the underground excavation and the geologic discontinuities in the immediate roof, such as bedding planes, faults and joints (Horne, Ferm and Currucio, 1978). Both of these causes will result in a zone of roof softening that influences the roof behaviour and controls the transverse loading pattern on the immediate roof. In a case study on South Africa collieries, a roof monitoring program using sonic extensometer suggested that a parabolic surcharge is loaded on the immediate roof by the formation of softened weak strata under the effect of sagging due to the lack of the nature support from beneath the strata (Canbulat and Van der Merwe, 2009), as shown in Figure 1.

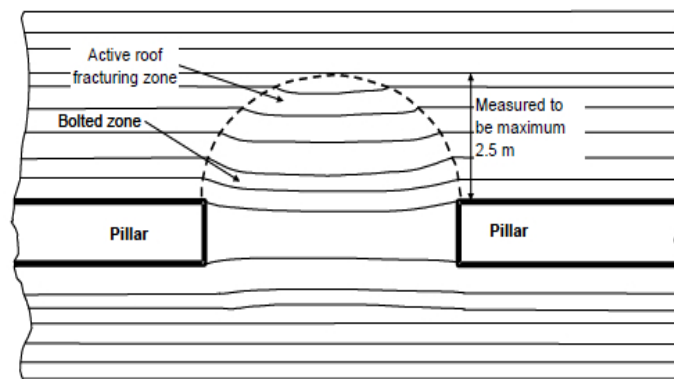


Figure 1: Roof behaviour model with zone of softening (Canbulat and Van der Merwe, 2009)

Roof support design approaches

The general idea behind the underground roof support is to reduce the magnitude of horizontal and vertical movements of the laminated strata by clamping them together and closing the separation of any pre-existing fractures that might have contributed by roof sagging after excavation (Hoek, Kaiser and Bawden, 1995). As there is no universally accepted design methodology in roof support system, many mines adopt an integrated methodology that combines the numerical, analytical and empirical methods. Three analytical failure modes, including shear, roof bolt tension and bond sliding are considered, with the design outputs back analysed by empirical modelling and geotechnical classification techniques (Canbulat, 2011). In these analytical models the aim is to ensure that the shear failure is prevented in the first place by installing sufficient number of roof bolts (i.e. reinforcement); however, if the shear failure occurs the roof should be stabilised in suspension mode, mainly using cables (i.e. post roof failure). These failure mechanisms can be classified under two well-known design models, namely, suspension and beam building, as shown in Figure 2.

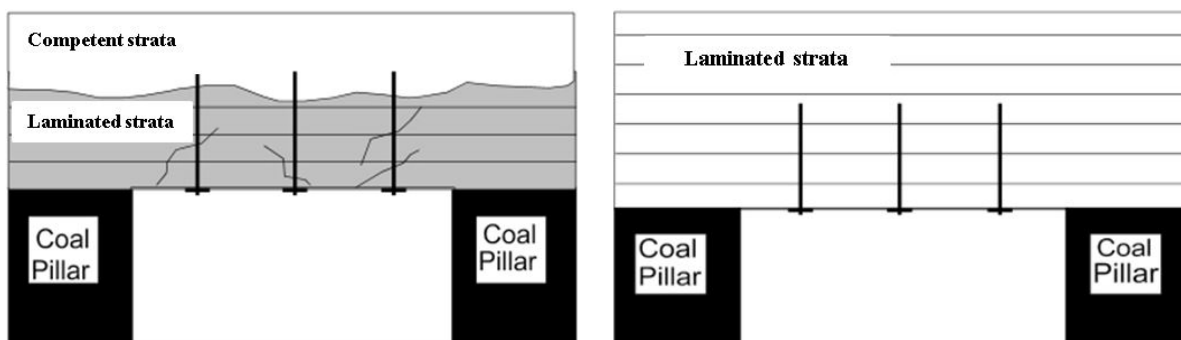


Figure 2: Analytical roof support models (After Mark, Molinda and Dolinar, 2001)
(Left: Suspension; Right: Beam building)

Suspension support mechanism is applicable to those situations where the immediate roof is comprised of weak strata or failed immediate (i.e. bolted) strata with stronger bedding or unfailed strata existing higher in the roof. The dead weight of the lower weak rock strata is suspended by using roof bolts or cables clamping these weak strata together and anchoring in the upper stronger strata. Two design criteria must be met the following conditions:

- The roof bolt or cable strength has to be greater than the weight of the loose or failed roof layers;
- The anchorage capacity of the support system is greater than the weight of the loose roof layers suspended; and

In shear failure mode it is assumed that the support mechanism is influenced by the interbedding shear stress induced under transverse loading and the shear resistance provided by the bolting system, including frictional resistance from bolt pre-tensioning and intrinsic shear strength of the bolts.

The details of these failure modes and the factors of safety against shear and suspension failures are given by Canbulat and van der Merwe (2009). It is of note that in this current study it is assumed that the load distribution across the beam is parabolic in all failure modes in order to achieve a consistent approach in calculation of loading on the roof support.

REVIEW OF STOCHASTIC MODELLING TECHNIQUES

Stochastic roof support design methodology

The underground roof support design is influenced by many elements, such as rock mass properties, mining geometries and bolting specifications. It is widely recognised that the rock mass properties can vary significantly within a short distance in a coal mine, leading to the roof stability to be considered as a random system where the occurrence of failure is a random event depending on the outcome of random variables involved (Chen, Jia and Ke, 1997). When compared with the traditional deterministic approach where single values are assigned to each design input, stochastic modelling technique has the advantage in dealing with the inherent uncertainties in the underground roof support design. The design inputs in the support system with the random values are represented as probability distributions, with the resulting factor of safety also expressed by a density function. Therefore, the associated risk of each support design can be quantified by calculating the area beneath the density function of FoS within a specified interval (i.e. less than unity). The following steps summarise the stochastic approach used in this paper in evaluating the roof support design:

- Select appropriate analytical models that produces a deterministic solution to the roof stability;
- Decide which input parameters are to be modeled probabilistically and the representation of their variability in terms of probability distributions;
- Repeatedly run the design output using the deterministic model by Monte Carlo simulation
- Obtain the probability density function of the design output (i.e. factor of safety) for each roof support design; and
- Evaluate the risks for each roof support design by considering the probability of failure, as illustrated in Figure 3.

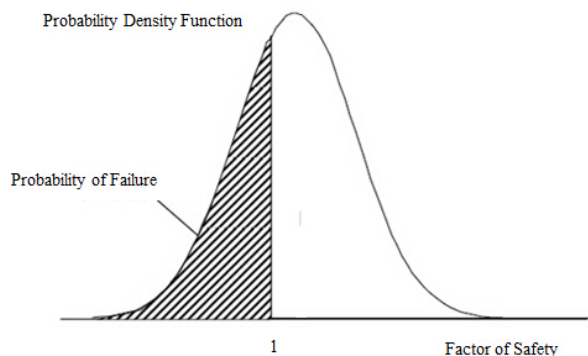


Figure 3: Risk representations by the probability distribution of factor of safety

Monte Carlo simulation

In order to obtain the density function of the design output, Monte Carlo Simulation is used to conduct a repeated deterministic calculation for a large number of times where single values of each input

parameters are sampled randomly from their dataset and each loop can produce a single value of design output (Rubinstein, 1981). The accuracy of the Monte Carlo simulation is related to the number of trials run, which is dependent on a variety of factors. Equation 1 shows the quantitative relationship between the number of simulation trials required and the desired confidence level in solution to the design output as well as the number of input variables included, based on the studies in civil engineering (Harr, 1987).

$$M_{mc} = \left[\frac{d^2}{4(1-\varepsilon)^2} \right]^m \quad (1)$$

Where:

- M_{mc} = number of Monte Carlo simulation trials
- d = standard normal z value corresponding to the confidence level
- ε = the required confidence level (0 to 100%)
- m = number of inputs variables

The number of Monte Carlo trials increases exponentially with the level of confidence and the number of variables. The factor of safety in bolt tensile failure contains three random variables (Canbulat and van der Merwe, 2009), the number of Monte Carlo simulation required is 309,445 under a desired confidence level of 90%. However, in the case of shear failure model seven random variables are involved, the number of Monte Carlo trials increases significantly to 6.8×10^{12} . Furthermore, with an increased confidence level to 95%, 1.2×10^{18} runs are required. Such large number simulations is extremely time consuming and therefore is not technically feasible for personal computers. The number of Monte Carlo simulation trials can be reduced by using Latin Hypercube Sampling (LHS). With this sampling technique, the entire space for each parameter is partitioned into an arbitrary number of dimensions and only one value will be selected within each dimension. The benefit of this sampling method is that it allows the value to be selected across the entire variable space and can be used to generate a representative distribution curve of a function of multiple variables with less sampling iteration (McKay, Beckman and Conover, 1979).

Goodness of fit tests

In the stochastic model, the randomness of the input parameters is accounted for by using appropriate probability distributions. Goodness of fit test is a broad class of statistical test that determines the best fit distribution model for each of the input parameters. It measures the compatibility of a random sample with a theoretical probability distribution function. The idea behind the goodness of fit tests is to calculate the value of a test statistic that measures the 'distance' between the actual data and the candidate probability distribution, and compare that distance to some threshold value. It is obvious that the probability distribution with the lowest test statistic value is considered as most compatible to the actual data sample. There are three common types of goodness of fit tests, Kolmogorov-Smirnov, Chi-square and Anderson-Darling tests. They differ in how the test statistics and critical values are calculated (Easyfit, 2014).

Chi-Squared test is used to determine if a sample comes from a population with a specific distribution. The main disadvantage of Chi-square test is that the sample data has to be binned and there is no optimal choice for the number of bins. Different formulas can be used to calculate this number based on the sample size (Harris and Kanji, 1983).

Kolmogorov-Smirnov (K-S) test is used to decide if a sample comes from a hypothesised (fully defined) continuous distribution. The main limitation of K-S test is that it gives more weight near the centre of the distribution than at the tails (Berry and Lindgren, 1996).

The Anderson-Darling (A-D) test is a general test to compare the fit of an observed cumulative distribution function to a defined cumulative distribution function. A-D test can be used to overcome the limitations of the other two tests mentioned above as it is applicable to both of the binned and unbinned data and also provide a more sensitive result at the tail region (Sinclair, Spurr and Admad, 1990).

Fundamentals of probability theory

For the joint probability between two events, two conditions need to be considered based on the dependency between them (Berry and Lindgren, 1996). If two events A and B are independent where the occurrence of any one does not affect the probability of the other, the joint probability of both of A and B to occur is given by multiplying the probability of events A and B, as shown in Equation 2.

$$P(A \cap B) = P(A) \times P(B) \quad (2)$$

If the probability of occurrence of event A is dependent on that of event B, the joint probability is then determined by:

$$P(A \cap B) = P(A) \times P(B | A) \quad (3)$$

Where $P(B|A)$ is defined as the probability of A to occur when B takes place.

Based on the assumed failure modes (and sequences), the overall probability of failure can be calculated using the above relationships. For the purpose of this study the overall probability of roof failure is calculated as follows:

Let A be the event that the roof fails in shear, and B be the event that the roof fails in suspension mode. The event B can only occur if shear failure has already occurred. Also, let B_1 be the event that the roof fails due to cable failure and B_2 be the event the roof fails due to weak bonding. If either B_1 or B_2 occur than B occurs. Based on these assumptions, the overall probability of failure, $\Pr(i)$ of any given support system can be calculated as follows (Stoklosa, 2014):

$$\Pr(i) = \Pr(A) [\Pr(B_1) + \Pr(B_2) - \Pr(B_1 \cap B_2)] \quad (4)$$

CASE STUDY

Input parameters in stochastic modelling

To evaluate the current roof support design at Mine A using stochastic modelling technique, a set of design inputs are selected based on the analytical support mechanism discussed above. These parameters include:

- Roadway width;
- Intersection span;
- Height of roof softening;
- Thickness of immediate weak strata;
- Unit weight of immediate weak strata;
- Roof bolt and cable pretension;
- Roof bolt ultimate tensile strength;
- Bond strength obtained from underground short encapsulated pull tests (SEPT);
- Coefficient of friction of laminated roof strata;
- Roof bolt, cable spacing;
- Roof bolt, cable length.

All of the above input parameters are to be expressed in probability distribution with a large field data set collected, except for roof bolt, cable spacing and length that will be modelled as single value applicable to the whole mine. In addition, some other variables are also collected in assessing the variation in underground geotechnical environment and characterising different geotechnical domains that may be subject to varied roof support strategies. Those parameters are as follows:

- Major horizontal principal stress;
- k-ratio (horizontal stress to vertical stress ratio);
- Young's modulus of overlaying roof at bolting horizon; and
- UCS of roof strata at bolting horizon.

Uncertainties in roof support design

A comprehensive understanding of the immediate roof is a critical component in the roof support design. Geological boreholes are used at Mine A in conjunction with geophysical logging to perform a detailed geological and geotechnical characterisation up to 10 m into the roof strata, which corresponds to the longest cable bolts available at the mine site and represents the highest bolting horizon. Geotechnical domains are then defined using these data, with the purpose of differentiating the roof support designs across zones with different geotechnical environment. The following geotechnical properties are discussed:

- Lithology, including thickness of laminated roof;
- Roof strata competency at primary bolting horizon (UCS, CMRR);
- Roof deformation; and
- Height of roof softening.

Immediate roof lithology

The case mine extracts the German Creek Seam within the Bowen Basin coalfield. The cover depth of current panels varies from 300 to 350 m. The lithology of immediate roof is characterised into five distinct roof zones based on the geophysical investigation, as shown in Figure 4.

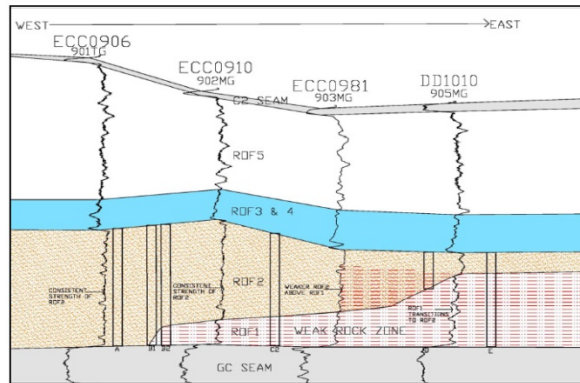


Figure 4: Sonic signatures in four boreholes (provided by the mine)

It can be seen that the weak, laminated roof, as represented by ROF1, has the potential to delaminate and soften under high horizontal stress and these roof zones are both critical to roadway performance and geotechnical design. Therefore, the thickness of the immediate weak rock strata can be used to define the geotechnical domains. To obtain a mine-wide profile of ROF1, geological data from 588 boreholes across the mine site is used and the contour map for the thickness of laminated roof is produced by using Surfer, as shown in Figure 5.

The lithological contour map in Figure 5 shows that the current panels that are in operation can be classified into two geotechnical domains based on the thickness of the laminated immediate roof. In the western panels from 901 to 904, the laminated roof varies from 0 to 1.2 m while in the eastern panels from 905 to 908 the thickness is consistently larger varying from 1.5 to 2.7 m. Therefore, different bolting strategies may be considered in these two geotechnical domains.

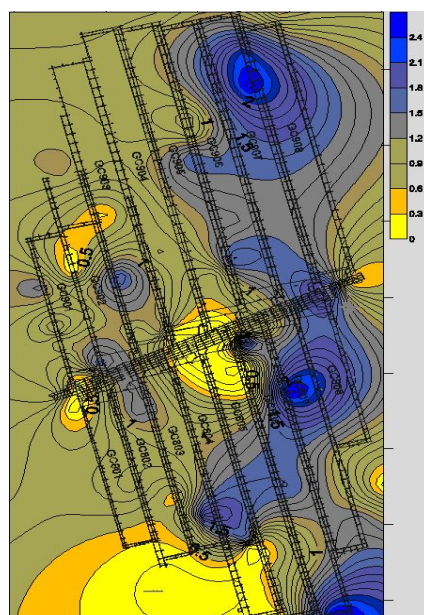


Figure 5: Contour map of thickness of ROF1 unit

The competency of the roof was assessed using the Coal Mine Roof Rating (CMRR) technique. Molinda and Mark (1994) suggest the following broad categorisation of roof competency:

- CMRR<45, weak roof;
- 45<CMRR<65, moderate roof; and
- CMRR>65, strong roof.

Based on an analysis of 588 boreholes at the mine's current workings, the roof strata at primary bolting horizon of 1.8 m has an overall CMRR of approximately 45, indicating that the roof competency can be generally classified as 'moderate'.

Roof deformation

Roof deformation at a total of 162 intersection and roadway measurement sites from 900s panels are shown in Figure 6. It can be seen that the magnitudes of roof deformation at intersections are consistently higher than those observed at roadways with the average being 6 and 18 mm at roadway and intersection respectively. With the proposed roadway and intersection span of 5.5 and 9 m respectively, the results indicated that for approximately 80% (as mined) per cent increase in the diagonal span at intersections relative to the roadway spans, on average, the magnitude of the displacement in the roof increased by approximately three times.

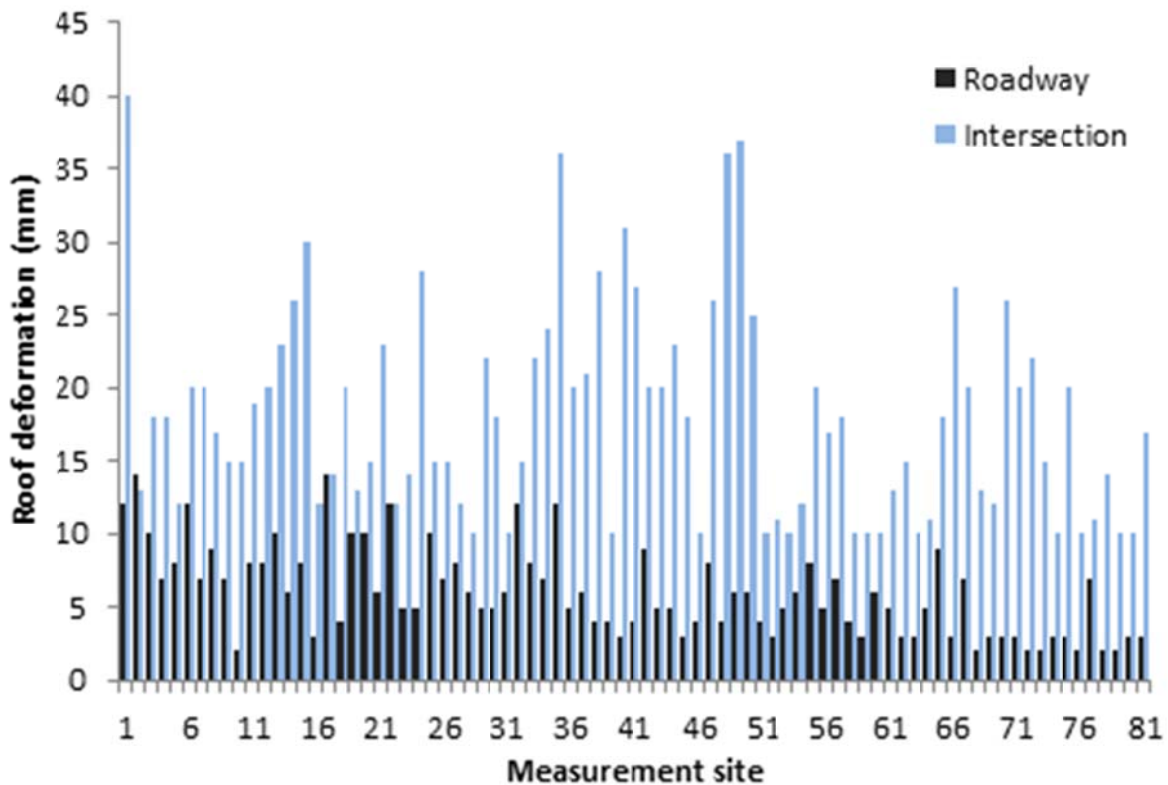


Figure 6: Measured roof deformation at roadway and intersections

Height of softening

As part of the roof monitoring program, 4-anchor tell-tales were used to monitor roof deformations once the primary supports are installed. For the purpose of this study, the height of roof softening is defined as the highest roof horizon where the deformation is larger than 2 mm. The data of height of softening obtained from 284 tell-tales at the mine site and the simulated continuous data using Monte Carlo trials is presented in Figure 7. The maximum measured height of softening is limited to 8 m into the roof, and there is no evidence of substantial roof instabilities beyond this height in the absence of the reported roof failure. The average HoS is approximately 2.7 m that is larger than the primary roof bolt length of 1.8 m currently used on site.

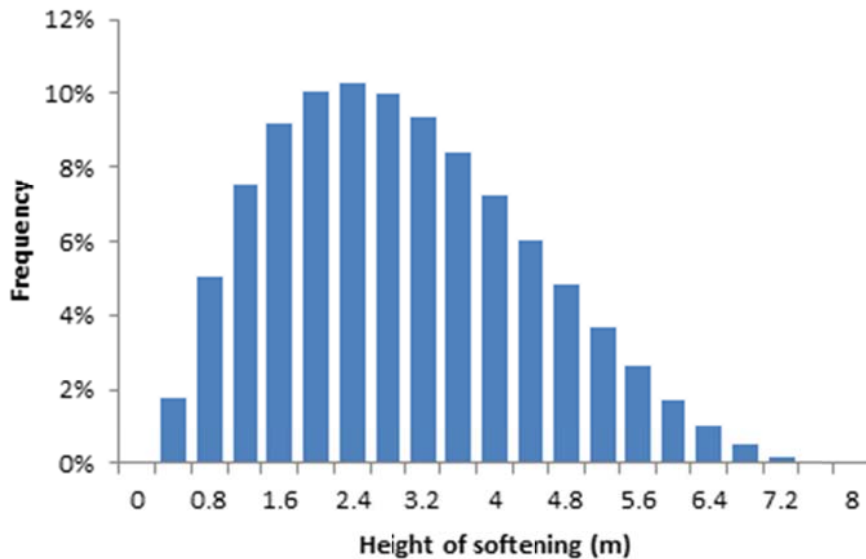


Figure 7: Measured height of roof softening

The roof deformation results show that the intersections are subjected to roof deformations almost three times of the roadways, which strongly suggest the necessity to divide the entire underground roof into two study areas: namely, roadways and intersections. The average HoS at roadway and intersection are 2.5 and 3.2 m respectively (excluding the cases with zero height of softening), as shown in Figure 8. It is evident from this figure that similar to the roof deformation measurements, the magnitude of HoS at intersections is larger on average than that in roadways, which is attributable by a larger roof span and also may indicate relatively higher risk of roof instability if insufficient roof support is installed in intersections.

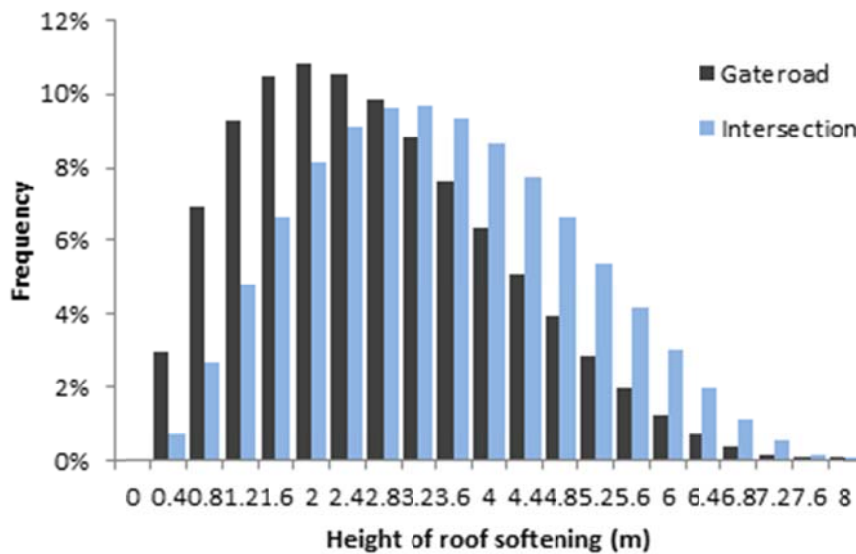


Figure 8: Subset of height of softening

Roof bolt and resin bond strength

The ultimate tensile strength tests are conducted for the primary roof bolts that are used in the current panels. Over 320 roof bolts with a nominal length of 1.8m have been tested. The results showed that the bolt ultimate tensile strength varied from 304 to 348 kN with an average value of 324 kN.

A series of underground short encapsulation pull tests were also carried out at the mine in various locations with relatively high strength of roof strata using 200 mm and 300 mm encapsulation of spin to stall and spin and hold resins using 1.8 m roof bolts currently being used by the mine. Roof bolt pull-out resistance demonstrates a high degree of variation, ranging between 0.25 and 1.18 kN/mm (maximum

load achieved in kN/encapsulation length in mm). The most likely cause of poor anchorage measured in these tests is varying rock competency. Some of the extremely low bond strength results are considered to be anomalies caused by resin losses and/or incorrect testing practices such as resin under or overspinning.

Bolt pretension

The pretension on the roof bolts can be estimated by the conversion from the torque measured during the bolt installation using a torque-wrench. Unfortunately, the relationship between tension and torque for fastened bolt is difficult to predict and in the real world variation as high as 30% can occur (Bickford and Nassar, 1998). Nevertheless, the installation audit reports indicated that the magnitude of pretensioning on the roof bolts varied from 33 to 75 kN.

PROBABILITY DISTRIBUTION OF INPUT PARAMETERS

Before the statistical determination of the best fit distribution fitting, it is necessary to conduct a preliminary study on the nature of data collected. Such process can screen out those candidate distributions that are explicitly not fit to the data set. This can help to narrow the choice to a limited number of distributions and save computational time, especially for those inputs with a large number of data points. The following factors of data set are considered:

- Data domain (continuous/discrete);
- Bound of data (fixed/open); and
- Negativity.

In general, specifying a fixed bound can enable the resulting distributions fitted to better reflect the randomness of the underlying data points. However, the number of candidate distributions decreases significantly if fixed bound is used otherwise. A preliminary analysis shows that for most of the design inputs the tail region of the proposed best fit distribution that is beyond the actual measured data range has a limited impact on the design output (i.e. on factor of safety and probability of roof failure). However, that is not the case for height of roof softening as the design outputs is very sensitive to the tail region in the proposed distribution. Without truncating the tail values the probability of roof failure will be significantly high, which is considered to be unrealistic. Therefore, for the purpose of this project, open bound is used for all of the design inputs, except for the height of roof softening, to introduce a larger candidate pool in the distribution fitting. The results of Anderson-Darling goodness of fit test are included in Table 1.

Table 1: Best fit distributions used in Monte Carlo simulation using Easyfit©

<i>Parameters</i>	<i>Samples</i>	<i>Min</i>	<i>Max</i>	<i>Ave</i>	<i>Distribution</i>	<i>Scale</i>	<i>Shape</i>	<i>Location</i>
Bolt strength (kN)	319	306	347	324	Lognormal	5.78 (μ)	0.03 (σ)	0
Bolt pretension (kN)	100	33.0	75	57.0	Weibull	61.3	6.25	0
Coefficient of friction	27	0.52	1.14	0.86	Weibull	0.92	5.33	0
Roof density (t/m^3)	115	2.34	2.70	2.54	Pearson5	44.5	96.9	0
Young's modulus (MPa)	90	2.2	28.9	9.25	Lognormal	1.80 (μ)	0.71 (σ)	1.47
Laminate thickness (m)	119	0.04	2.5	0.86	Erlang	0.43	2	0
HoS – Roadway (m)	112	0	8.0	1.93	Pert	0 (min)	8 (max)	1.80 (mode)
HoS – Intersection (m)	172	0	8.0	3.42	Pert	0 (min)	8 (max)	2.85 (mode)
Major stress (MPa)	61	5.93	22.8	12.0	Gamma	1.16	10.4	0
Roadway width (m)	1136	4.72	7.87	5.48	Pearson5	1502	275	0
Intersection span (m)	257	7.35	11.9	9.93	Weibull	10.28	14.6	0
Bond strength (kN/mm)	17	0.25	1.18	0.83	Gamma	0.04	27.7	0

It should be noted that the some of the results (e.g., interbedding coefficient of friction, bond strength) presented in Table 1 are based on a limited number of data points and/or the limits of the software utilised to conduct the Monte Carlo simulations. Therefore, the best fit probability distributions of these design inputs obtained from GoF tests are only marginally better than the others.

RISK-BENEFIT ANALYSIS OF ROOF SUPPORT DESIGN

Required number of Monte Carlo simulation

A sensitivity analysis is conducted to investigate the optimum runs of Monte Carlo trials with the main objective to determine the minimum number of Monte Carlo trials from which further increasing the trials will not significantly improve the computational performance. In order to achieve this, the probability of roof failure under shear failure mechanism is calculated 10 times with various numbers of trials and the variation in the results are compared. The results are presented in Figure 9. The computed PoF from the Monte Carlo simulations becomes insensitive to the number of trials after 100,000 trials, implying potentially an optimum trial runs for the analytical models used in this study.

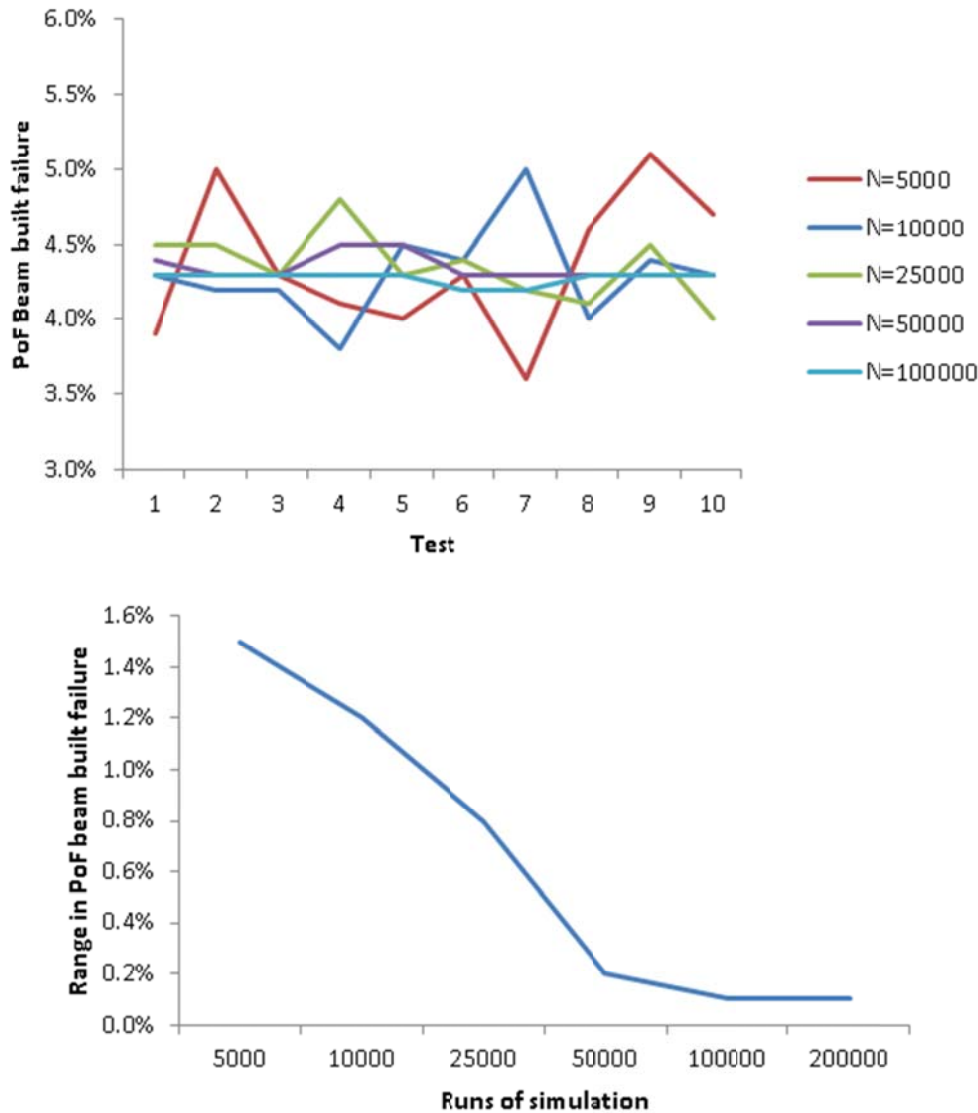


Figure 9: Sensitivity analyses on optimum Monte Carlo trials

Probability of failure and roof support design

A risk-benefit analysis of roof support design involves the evaluation of the overall probability of roof failure that incorporates three types of failure modes, including shear failure, bolt tensile and bond sliding failure in suspension mechanism. Individual probability of failure of these failure modes is determined by calculating the area under the density curve of safety factor <1 from a total of 100,000 Monte Carlo simulation trials. The mine's current bolting density is used in the analysis. 4 or 6 roof bolts are installed in a row with row spacing of 1 m. The cables are in a bolting pattern of 2 bolts in a row with 2 m row interval.

Roadway roof support design without cable bolts

Table 2 summarises the probabilities of stabilities for 6 roof bolt patterns with 1 m row spacing. It is evident that increased roof bolt length will reduce the probability of failure in all three failure modes. In general, the risk of roof failure under suspension supporting mechanism can be reduced by using longer roof bolts. However, even with the longest 2.4 m roof bolt, the resulting PoF can still be as high as 53%. The possible reason is that the analytical model of suspension mechanism is only considered effective when the height of roof softening does not exceed the bolt length. For any roof strata with height of softening beyond bolting horizon, a safety factor of zero is assumed. Therefore, cable bolts are required to reinforce the roof zone with a relatively high elevation of softening in suspension mechanism.

Table 2: Probability of failure in a roadway with currently used roof bolt densities

Bolting pattern	PoF	Roof bolt length (m)	
		1.8	2.1
6 bolts in a row with 1 m row spacing	Shear loading	0.045%	0.002%
	Bolt suspension failure*	67.6%	59.5%
	Bond suspension sliding*	68.4%	60.4%
	Overall	0.040%	0.002%

* calculated only for the heights of softening that are less than the roof bolt length

Roadway roof support design with cable bolts

As indicated above, although the overall probability of roof failure can be significantly reduced by using longer roof bolts, the suspension failure modes still remain the main sources of roof instability. In order to mitigate such risks, cable bolts are recommended. Bolting patterns that consist of 1.8 m roof bolts with six bolts in a row at 1.0 m row spacing and cable bolts in varied lengths of 4, 6 and 8 m with two cables in a row at 2.0 m row spacing are evaluated in suspension and shear failure mechanisms. The resulting PoF for each bolting plan is presented in Table 3.

It is evident in this table that that the risks of roof failures in all failure modes are substantially reduced when cable bolts are introduced. In addition, the PoF of bolt tensile and bond sliding failure can be decreased by increasing the length of cable bolts from 4 m to 6 m. Further increase in the cable length will not benefit the roof stability substantially. Figures 10 and 11 present the distributions of factor of safety for the roof support design with a combination of 1.8 m roof bolt and 6 m cable bolt, based on which PoF is calculated.

Table 3: Probability of failure in a roadway with currently used 1.8m long roof bolt and cable densities

Bolting pattern	PoF	Cable bolt length (m)			
		Roof bolts only	4	6	8
Six 1.8 m roof bolts, 1 m row spacing with two cable bolts, 2 m row spacing	Shear loading	0.045%	~0	~0	~0
	Bolt suspension tensile*	67.6%	18%	1.8%	1.3%
	Bond suspension sliding*	68.4%	23%	3.4%	0.029%
	Overall	0.040%	~0	~0	~0

* calculated only for the heights of softening that are less than the cable lengths

Intersection roof support design

Roof strata at intersections is expected to have a higher risk of failure due to inherently larger spans, higher levels of deformations and height of softening, as shown in Figures 6 and 8. The roof span is defined as the average diagonal width of the intersection and the required support density is calculated for this length. A preliminary study showed that the minimum PoF at intersection that can be achieved by using 2.4 m roof bolts solely is approximately 2%, which is significantly higher than the PoF in roadways. Therefore, cable bolts are required to reinforce the roof strata at intersections. Table 4 summarises the probability of failure of the current intersection support with cables. Similar to the cases in roadway support, introducing additional cables can significantly improve the roof stability by reducing the bolt tensile and sliding failures. The probability of failure in shear failure mode can also be reduced by installing cables. However, as the cables increase the shear capacity only in the bolted horizon, an increased in cable length will not result in reduced PoF in shear failure mode.

Table 4: Probability of failure in an intersection with currently used cables

<i>Bolting pattern</i>	<i>PoF</i>	<i>Cable bolt length (m)</i>			
		<i>0</i>	<i>4</i>	<i>6</i>	<i>8</i>
Twelve 1.8 m rock bolts, 1 m row spacing with two cables, 1 m row spacing**	Shear loading	10.6%	2.5%	2.4%	2.4%
	Bolt suspension tensile*	81.2%	32.7%	4.5%	1.5%
	Bond suspension sliding*	81.8%	39.2%	8.2%	0.1%
	Overall	10.25%	1.50%	0.30%	0.04%

* calculated only for the heights of softening that are less than the cable lengths

**twelve 1.8 m rock bolts as counted across the intersection diagonal width

Exposure and quantitative risk

In the above section, a stochastic approach was demonstrated to calculate the probability of failure with various roof support design. However, a probability of failure alone is not useful unless it is used to calculate the quantitative risk by combining the consequence in the consideration (i.e. Risk = PoF × Consequence). In this section an example is given, with the aim of calculating the annual probability of having a fatal accident across an underground working section. A similar concept for South African gold mines was also published by Stacey and Gumede (2007) and utilised in this paper.

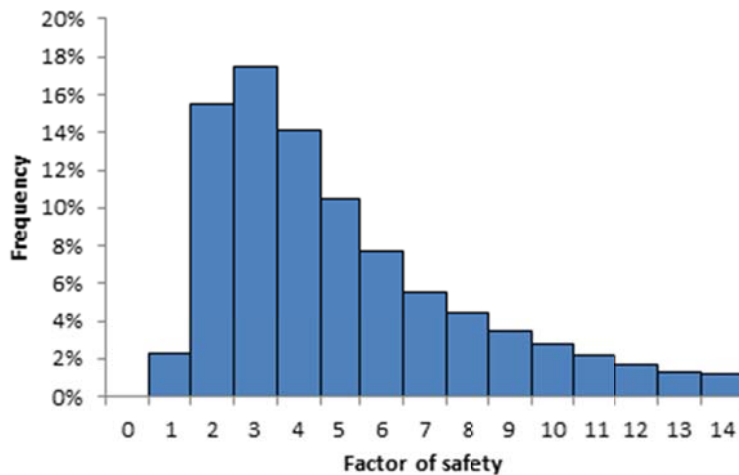


Figure 10: Distribution of factor of safety in shear failure mechanism (1.8 m roof bolt/6 m cable)

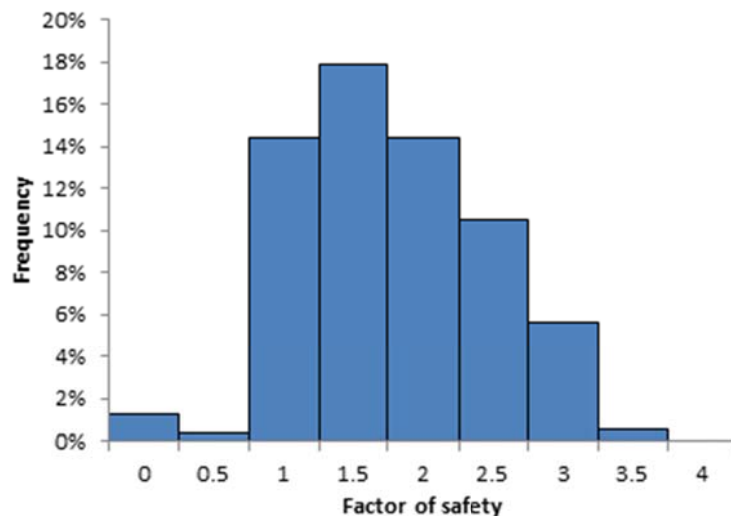


Figure 11: Distribution of factor of safety in suspension failure mechanism (1.8 m roof bolt/6 m cable)

For a double heading gateroad with 100 m long chain pillar and 5.5 m roadway width, the total area of exposure is 1,100 m². The roof support used in this section is assumed to be 6×1.8 m long roof bolts installed at 1 m spacing interval. Assuming the following:

- There are 6 miners in this panel area for a period of 12 hours during the day and night shifts;
- Each miner expects to occupy an area of 1 m²;
- The work within this panel area is scheduled for 30 days a month and 12 months a year;
- The probability of roof failure with the assumed roof support is 0.036%; and
- The roof fall will result in a fatality.

The total annual exposure hours can be calculated as follows:

Total hours per year = 365 days × 24 hours = 8,760 hours

Total shift exposure = 12 months × 30 days × 24 hours = 8,640 hours

Panel area = 100 m × 5.5 m × 2 = 1,100 m²

Area occupied by miners on day/night shifts = 6 miners × 1 m²

Probability of annual occurrence of roof fall fatality = $(2 \times 8,640 / 8,760 \times 6 / 1,100) \times 0.04\% = 3.8 \times 10^{-6}$

The results calculated above indicate that, with a six roof bolt pattern, the risk of a fall of ground fatality is approximately 4 in 1,000,000 employees or 1 in 250,000 employees in all ground conditions, i.e., without a TARP based pro-active strategy. Since the mine uses a comprehensive TARP system, it is considered that the actual probability of failure should be lower than this number as additional roof bolts are installed by triggering the TARP in the case of deteriorating ground conditions. Nevertheless, the acceptability of this roof support design can be evaluated against the relevant design criteria. The acceptable fatality rates have been proposed by various publications. Wong (2005) states that risks which have a fatal injury rate of 10⁻⁵ or more are unacceptable. Terbrugge *et al.* (2006) and Steffen and Terbrugge (2004) suggested the use of internationally accepted design criteria that proposed an annual probability of fatality of 1 in 10⁻⁴.

CONCLUSIONS

The data collected as part of this study confirms that the rock mass properties and support-rock interface exhibit high degrees of variations. These variations should ideally be quantified using probability distributions in a roof support design. Using the Anderson-Darling goodness of fit test, best probability distributions for various input parameters have been identified. The results indicated that specifying a fixed bound can enable the resulting distributions fitted to better reflect the randomness of the underlying data points. However, the number of candidate distributions decreases significantly if fixed bound is used otherwise. A preliminary analysis also indicated that for most of the design inputs the tail region of the proposed best fit distribution that is beyond the actual measured data range has a limited impact on the design output (i.e. on factor of safety and probability of failure). However, the results revealed that the design outputs is highly sensitive to the tail region in the proposed distribution in the case of height of softening. Without truncating the tail values the probability of roof failure will be significantly high, which is considered to be unrealistic. Therefore for the purpose of this project, open bound is used for all of the design inputs, except for the height of roof softening, to introduce a larger number of possible distributions.

An attempt has also been made to demonstrate the significance of quantifying the risks associated with roof failures. Three failure mechanisms, namely shear failure, bolt tensile failure and bond failure, have been used in this study. The results revealed that in general, the highest probability of failure is associated with bolt tensile failure and bond sliding from dead-weight loading if rock bolts are used solely in roof reinforcement, implying the necessity of longer roof bolts and/or cables. Analysis results indicated that both of the risks associated with bolt tensile and bond sliding failures can be significantly reduced by additional cables with increased bolt length, resulting in an improvement in the overall roof stability.

Various roof support designs with a combination of roof bolts and cables were investigated for roadway and intersections, based on which an example was demonstrated to assess the quantitative risk of roof fall fatality for an underground working section by considering both of the probability of roof failure and underground workforce exposure.

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